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Volume LIII, No. 8. LONDON, AUGUST, 1958.

EDITORIAL NOTES

The Durability of Concrete.

A FURTHER stage has been reached in the long-period tests of concrete started in the year 1941 by the American Concrete Institute. The latest tests and observations were made in the year 1955, and confirm previous tests in showing the value of air-entrainment in improving the resistance of concrete to freezing and thawing and other climatic conditions. The cements used included ordinary and rapid-hardening Portland, low-heat, sulphate-resistant, and moderate sulphate-resistant, to some of which were added air-entraining agents. After twelve to fourteen years of exposure the amount of deterioration due to weathering in the milder and the moderate-to-severe climates was not sufficient to permit comparisons to be made of the performance of the different cements used, as there was little visible evidence of deterioration in any of the structures, although there was slight deterioration of some of the laboratory-made specimens stored at the sites of the structures in which similar concrete was used. In the case of some of the structures subjected to severe conditions, however, some comparisons could be made.

One of these is a road subject to heavy traffic, heavy falls of snow, and rapid changes of temperature, and partly on a gradient which was regularly treated with a chemical to remove ice. Twenty-seven different types of cement were used in slabs 75 ft. long. There was no flaking of the surface of the parts of the gradient in which air-entraining agents were used, whereas flaking occurred on up to two-thirds of the area of the slabs without air-entrainment. On the level parts of the road only about 5 per cent. of the surface of the concrete without an air-entraining agent had flaked. The type of cement used seems to have little effect so far as flaking is concerned, although it is thought that low-heat and sulphate-resistant cements gave better results than the other cements used. Except on the gradient there was extensive cracking throughout the road, but this seemed to be related to the proportion of cement rather than to the type of cement.

At Cape Cod, sixty piles 12 in. square by 30 ft. long were driven in sea-water in the year 1941 and have since been subjected to alternate freezing and thawing. The piles project slightly above water at high tide, and the tidal range is about 9 ft. Three classes of concrete were used, namely 470 lb. of cement per cubic

yard and a slump of 2 in., and 660 lb. of cement per cubic yard with slumps of 2 in. and 8 in. The piles made with the lean mixture are in the worst condition. Some of the piles contained an air-entraining agent, and these were all in better condition than piles made with the same mixtures but without air-entrainment.

Columns, ground slabs, and tanks were cast in 1941 and stored in the open in severe climatic conditions. In 1955 a report was made on the condition of the tanks. These are 30 in. cubes; the bottoms are 15 in. thick, and the walls are 4 in. thick at the top tapering to 8 in. where they meet the bottom. The space so formed has been kept filled with sand and water. The proportions of the concrete mixtures are from 425 lb. of cement per cubic yard with a slump of 8 in. to 610 lb. of cement per cubic yard and a slump of 1½ in. In the case of the richer mixtures containing rapid-hardening, low-heat, and sulphate-resistant cement there has been less deterioration than in the case of specimens made with ordinary Portland cement and moderate-sulphate-resistant cement. In the case of the leaner mixtures there has been much deterioration and complete failure of some of the tanks made with ordinary and rapid-hardening Portland cement, whereas three of the four specimens made with low-heat cement are still in good condition. On the other hand, all except one of the specimens including an air-entraining agent were still in perfect condition, including those with only 425 lb. of cement per cubic yard and with a slump of 8 in.

The effects of exposure of these structures to the weather, particularly those subjected to severe freezing and thawing, have been compared with laboratory work on the composition of the cements and the testing of specimens made with similar cements. It was found that there was no relationship between the heat of hydration of the cement and the performance of the concrete, although there were indications that concrete made with low-heat cement had improved resistance to freezing and thawing. Apart from low-heat cements, which have low contents of C_3S and C_3A, no relation was found between the C_3A content of the cement and the durability of concrete. Except in cases in which alkali-reactive aggregates were used, the alkali content of the cement had no effect on the durability of concrete; the total alkali content of the cements varied from 0·23 to 1·36 per cent. The fineness of the cement seemed to have no effect on the durability of the concrete. Specimens made with low-heat cement, the fineness of which was between that of ordinary and rapid-hardening Portland cements, were in the best condition, but it is thought that some other factor may have been responsible for the better performance of the low-heat cements. Compressive and bending tests on specimens made in a laboratory indicated considerable differences in the strengths of the cements used, but so far these differences seem to have had no effect on the durability of the concrete. Some of the cements had a tendency to stiffen shortly after the concrete was mixed; it is known that when such cements are used concrete will regain its plasticity with prolonged mixing and will then harden normally, and the tests show that cements with a false set had no harmful effect on the concrete. Tests of specimens made in a laboratory showed that the contraction of neat cement prisms was greater in the case of cements with an air-entraining agent, but this had no effect on the performance of the test structures. Freezing and thawing tests of specimens made in a laboratory showed marked superiority of the cements with air-entraining agents, and this was confirmed by the outdoor tests.

Losses of Prestressing Force.

By PAUL W. ABELES, D.Sc. (Vienna), M.I.Struct.E.

THE methods of analysis and design of prestressed concrete generally used in Great Britain are based on the assumption that the prestressing force applied to the concrete (which is also termed the prestressing force " at transfer "), and the minimum effective prestressing force acting on a section, are both known. In a simply-supported beam, for example, an effective eccentric compressive force of definite magnitude is taken into account, and the stresses due to this force are directly superimposed on those due to the loading, the whole system being assumed to remain statically determinate. Although this method is not entirely exact, numerous tests have proved that actual behaviour conforms with sufficient accuracy to these assumptions.

An " exact " method is often used, for example in Germany, which allows for the fact tnat a simply-supported beam becomes statically indeterminate when it is prestressed, the initial effect of the prestressing force and the effect of changes occurring with the passage of time being therefore considered independently of each other. Although this may be theoretically correct, its application to practical design produces complications. The designer must know in advance the conditions under which the concrete will be cast and cured, and must also allow for the effect of weather conditions on creep. It is rarely possible for these conditions to be accurately anticipated, and it is essential that the most unfavourable assumptions be made in order to allow for the maximum possible losses of the initial prestress. The accuracy of the method is therefore unlikely to exceed that of the simpler method.

If the design is based on the minimum effective prestressing force, it is essential to determine the maximum possible losses of prestress which the various influences can cause. When these are known the calculation is much simpler than, and quite as accurate as, a detailed computation in which each influence is investigated separately. A more detailed calculation may sometimes be worth while when the actual deformation of a structure is required to be known at a definite time or when the conditions at a certain time are of special interest (for example in the case of the loading of a test specimen), but even in this case a simple design with an additional eccentric force included to allow for the reduction in losses is usually adequate.

The minimum effective prestress occurs when maximum losses are assumed to occur after the prestressing force is applied to the concrete, and it follows that the worst conditions for the factors governing the losses should be assumed unless any particular factor is definitely known to be more favourable. For example, the magnitude of creep increases as the stress in the concrete increases. The loss of prestress as a result of creep therefore varies considerably with different stresses in the concrete, and it is clearly incorrect to adopt a constant overall value for losses (for example 15 per cent.) which disregards the magnitude of the compressive stress in the concrete adjacent to the prestressing steel. On the other hand, the concrete may be moist cured for the whole of the time until the member is placed in position, and in such an unusual case little creep need be considered for the first stage when the stress in the concrete adjacent to the steel is high. It is also possible that in some special cases the construction when

in use will always be exposed to moist surroundings, and in this case shrinkage and creep will be very small. In general, however, more unfavourable conditions must be considered.

The factors that lead to losses of prestress are discussed in the following, and suitable values are suggested.

Loss of Stress between the Jack and the Anchorage.

Post-tensioned steel may be anchored in three ways. When wedges are used, some slipping between the wires and wedges is unavoidable when the tensile force applied to the steel by means of the jack is transferred to the wedges. When anchor-plates are used in conjunction with nuts and threads, or the steel forms loops around the concrete, a small shortening of the steel may occur due to the anchor-plate or loop being pressed into the concrete. All these factors should be allowed for at the time when the tension is applied to the steel; similarly, any errors in calibration, and any losses due to friction in the connection between the pump and the dial-gauge, should be determined and allowed for at the time of tensioning by increasing the force applied by the jack. It is essential that there be approximate agreement between the measured elongation of the steel and the stress corresponding to the force indicated by the gauge.

The designer need consider these losses only when wedges are used to anchor short wires. A slip of the order of $\frac{1}{8}$ in. usually occurs at a wedge. The extension of steel with a modulus of elasticity of $28{\cdot}8 \times 10^6$ lb. per square inch due to a stress of 144,000 lb. per square inch is 6 in. in 100 ft., and a reduction of $\frac{1}{8}$ in. in this extension corresponds to a loss of one forty-eighth, or 3000 lb. per square inch, of the initial stress in the steel. This is easily allowed for by adding 3000 lb. per square inch to the initial tensioning stress. Such losses may occur with both pre-tensioned and post-tensioned steel. If the steel were only 10 ft. long, however, a shortening of $\frac{1}{8}$ in. would correspond to a loss of prestress of 30,000 lb. per square inch, and this is far too much to add to the specified initial tensioning stress. This loss should be allowed for in the design, and a lower tensioning stress specified for any steel secured by wedges when its length is less than 30 ft.

When preparing the specification and drawings, it must be made clear which losses must be allowed for at the site by increasing the initial tensioning stress, and that the specified prestressing force is that which is to be applied when allowance has been made for these additional losses.

Elastic Shortening.

(1) Pre-tensioned Steel.—When the steel is pre-tensioned a reduction of the initial stress in the steel occurs at transfer since, as soon as the concrete is compressed, an elastic shortening of the concrete takes place. This is accompanied by an equal reduction in the length of the steel, with a consequent reduction in the initial prestress, and is given by

$$\frac{f_{sT}}{E_c} = \frac{Lp_e}{E_s} \qquad (1)$$

in which Lp_e is the loss of the initial tensioning stress in the steel due to the elastic shortening of the concrete, f_{sT} is the stress in the concrete adjacent to the steel

and E_c and E_s are the moduli of elasticity of the concrete and steel; the ratio $\frac{E_s}{E_c} = m$. If the compressive stress in the concrete adjacent to the steel is known, (1) becomes

$$Lp_e = m.f_{sT} \qquad (2)$$

In general the compressive stress f_{sT} is unknown, since it depends on the prestressing force at transfer (P_t), which in turn depends on the loss due to elastic shortening. In this case

$$f_{sT} = \frac{P_t}{A} + \frac{P_t.e_s}{\frac{I}{e_s}} = \frac{P_t}{A}\left(1 + \frac{e_s^2}{r^2}\right) \qquad (3)$$

in which A is the area of the concrete section and $r^2 = \frac{I}{A}$. Also

$$P_t = A_{st}(p_i - Lp_e) \qquad (4)$$

in which A_{st} is the area of tensioned steel. Therefore, combining (1), (3), and (4),

$$\frac{Lp_e}{E_s} = \frac{(P_i - Lp_e).A_{st}}{E_c A}\left(1 + \frac{e_s^2}{r^2}\right) \qquad (5)$$

from which

$$Lp_e = \left(\frac{m.K_e}{1 + m.K_e}\right)P_i \qquad (6)$$

in which

$$K_e = \frac{A_{st}}{A}\left(1 + \frac{e_s^2}{r^2}\right)(\text{see } Fig.\ 1) \qquad (6a)$$

Expression (2), however, can generally be used with sufficient accuracy even when the stresses in the concrete at the time when the prestress is applied are only

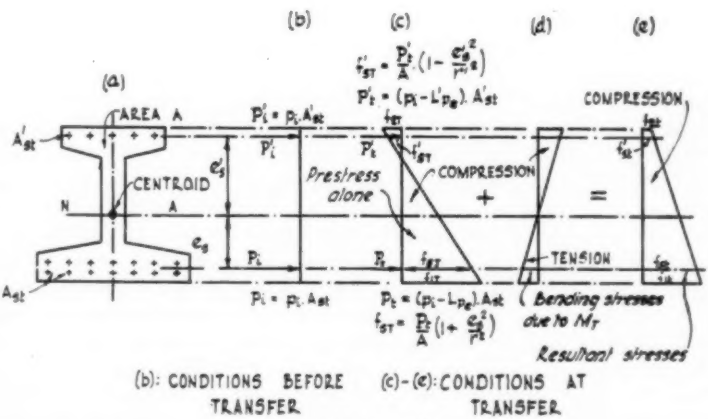


Fig. 1.—Elastic Losses with Pre-tensioned Steel.

assumed, as the stress f_{sT} in the concrete adjacent to the steel may then be estimated with sufficient accuracy from the maximum compressive stress. If full agreement between the assumed and actual values of f_{sT} is not obtained at the first attempt, the improved value arising from the first attempt should be inserted in equation (2) and the calculation repeated. The ratio between f_{sT} and the maximum compressive stress in the concrete depends on the arrangement of the steel in the cross section, and may be determined exactly when the position of the steel is known.

When an eccentric prestressing force produces a bending moment which opposes that due to a downward load, a concrete member supported at its lower surface will deflect upwards and the dead weight at the time of prestressing (that is at transfer) will then produce a bending moment M_t of opposite sign to that due to the prestressing force. The stresses then developed will be the resultant of those due to the prestressing force at transfer (denoted by the subscript $_T$) and those due to the bending moment M_t (the resultant being denoted by the subscript t). Consequently, this stress might include the tensile bending stress;

for example,
$$f_{1t} = f_{1T} - \frac{M_t}{\frac{I}{e_1}}.$$

It is sometimes stated, incorrectly, that loss at transfer due to elastic shortening is offset by elastic elongation when the member is loaded. The elongation of both steel and concrete which occurs in the tensile zone when a member is bent is accompanied by tensile bending stresses which reduce the precompression of the concrete. The effective elongation of the steel (related to zero extension of the concrete) is unaffected by any combined elongation of steel and concrete, and the loss of stress can be caused only by the shortening of the steel due to the elastic shortening of the concrete at transfer. This is given by equation (2), which is based on the stress f_{sT} in the concrete adjacent to the steel that is produced by the prestress alone; the resultant stress f_{st} must not be used. It is therefore immaterial whether lifting of the member occurs at the instant of prestressing or whether it is delayed, as the combined elongations due to bending cannot affect the magnitude of the loss due to elastic shortening.

(2) Post-tensioned Steel.—In the case of post-tensioned steel there is no loss of stress due to elastic shortening when the whole of the prestress is applied to the concrete in one operation, as the steel is not anchored until the complete prestressing force is obtained, by which time the elastic deformation of the concrete has taken place. If all the steel is not tensioned at the same time, however, tensioning of each steel member or group of members after the first produces an elastic deformation which modifies the length of, and the stress in, every anchored steel member, and losses due to elastic shortening at each operation are obviated only in the steel member or group of members tensioned during the operation. Consequently, if equal tensile forces are required in each steel member, the initial forces in each must be adjusted to allow for subsequent losses due to elastic shortening.

When the order of tensioning each steel member or group of members is known, it is not difficult to evaluate the loss of stress in each due to elastic shortening. The calculation is frequently tedious, however, and a simplified method is

available. This is based on the general condition that the average elastic shortening of every steel member will never exceed one-half of the elastic shortening which would occur if all the members were pre-tensioned ; equation (7) therefore gives a safe approximate value, although the stress in each member differs from the mean value so obtained. Strictly speaking, the effect only of the prestress at transfer should be considered, but the stress f_{st}, which includes the stress due to the bending moment M_t, may appropriately be used in this case as the counteraction of M_t is gradually offset as each steel member is tensioned.

$$Lp_e \text{ (post-tensioning)} = \tfrac{1}{2}.m.f_{st} \quad . \quad . \quad . \quad . \quad . \quad (7)$$

(3) General Considerations (see *Fig.* 2).—Suitable values for the modular ratio m have been discussed previously[1] ; the values given in *Table* I are based on the relationship between the cube strength at transfer and the modulus of elasticity for concrete as given in the British draft Code, and on a modulus of elasticity for steel of 29×10^6 lb. per square inch. If tensioned steel is provided

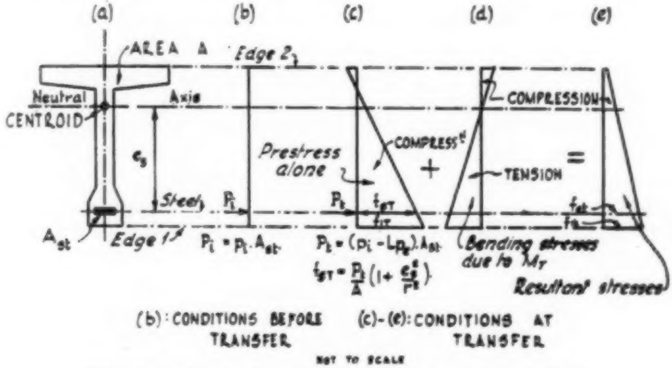


Fig. 2.—Elastic Losses with Post-tensioned Steel.

only in the tensile zone the losses due to elastic shortening may be calculated with sufficient accuracy by adopting a value for f_{sT} equal to the stress in the concrete at the centroid of the tensioned steel, and assuming that the loss is the same for each wire or bar. If the steel is distributed throughout the section, however, or separate tensioned steel is provided at the top and bottom, this assumption is not justified, and should not be employed.[2] The resultant stress in the concrete adjacent to the centroid of the steel will then be $f_{1t} = f_{1T} - \dfrac{M_t}{\frac{I}{e_s}}$.

It is obvious that the tensioned steel which prestresses the concrete cannot take part in resisting the prestress. Consequently the values of A, I, and e_s in equations (3), (5), and (6*a*) should be related to the net cross-sectional area, that is the area of the concrete less the area of the tensioned steel (in the case of post-tensioning the area of the ducts) plus $(m - 1)$ times the cross-sectional area of any untensioned steel. On the other hand, when calculating $\dfrac{M_t}{\frac{I}{e_s}}$ the entire area

should be considered by adding $m.A_{st}$ to the area of the cross section. Generally this distinction is unnecessary in design, particularly with pre-tensioned steel, as the difference between the area of concrete A_c and the net area

$$A_o = A_c - A_{st} + (m - 1).A_{su}$$

is usually negligible. Similarly, the difference between A_c and

$$A_e = A_o + m.A_{st}$$

is usually very small. With post-tensioned steel, however, the net area A_o may be reduced by the area of the ducts and this should be taken into account by reducing the area A_c by the sum of the areas of all the ducts. Obviously, if test results have to be analysed it is essential to consider in equations (3), (5), and (6*a*) the correct cross-sectional area A_o and not A_c.

Losses Due to Shrinkage and Creep of Concrete.

The nature and effect of shrinkage and creep have been considered previously[1] and the following notes concern the maximum losses arising from these phenomena that should be allowed for in design.

With pre-tensioning little or no shrinkage takes place before transfer when moist curing is carefully employed, and it should therefore be assumed that the full shrinkage occurs after transfer. A smaller loss is frequently allowed for when post-tensioning is employed, on the assumption that an appreciable amount of shrinkage occurs before the prestress is applied. While this may be so, however, it is preferable to assume that satisfactory curing so reduces the shrinkage that the full amount may be considered to occur after transfer even if this results in a slight reduction of the effective prestressing force. If this is achieved the reduction is offset by the fact that the entire tensile resistance of the concrete remains available when preliminary shrinkage cracks are prevented by careful curing, although this may be difficult to ensure for members more than 100 ft. long.

The ultimate shrinkage, under the worst conditions, may be assumed to be 0·03 per cent., which corresponds to a reduction in the tensioning stress of $0{\cdot}3 \times 10^{-3}.E_s$ lb. per square inch. When $E_s = 30 \times 10^6$ lb. per square inch, this amounts to 9000 lb. per square inch. With post-tensioning the loss may be reduced to $0{\cdot}2 \times 10^{-3}.E_s$, that is 6000 lb. per square inch, when some shrinkage has occurred before prestressing. It should be noted that these values are appropriate for a temperate climate such as that of Great Britain, but shrinkage may be greater in hot and dry conditions, or less in a permanently humid climate.[1]

The amount of shortening, and therefore the losses due to creep, depend on the ratio of the compressive stress to the strength, and this ratio will vary with time. Attempts have been made to solve this problem mathematically, the exponential formula for the total losses due to creep being obtained by integration. However, such an analysis ignores the fact that the effective prestress is decreased not only by creep but also, and at the same time, by shrinkage and relaxation of the steel. Consequently, the analytical formulæ based on reduction due to creep alone are not correct, and it is quite as accurate to assume that the mean compressive stress from which the total creep may be calculated is 85 per cent. to 90 per cent. of the stress in the concrete adjacent to the centroid of the tensioned steel. For members with pre-tensioned steel the British draft Code of

Practice for Prestressed Concrete recommends a maximum allowance for creep of $0{\cdot}33 \times 10^{-6}$ for a stress of 1 lb. per square inch in the concrete adjacent to the steel. For an average stress of 1000 lb. per square inch applied at the centroid, the losses due to creep would exceed those due to shrinkage by 10 per cent.; an allowance should, however, be made for the reduction in stress which occurs due to the other losses, and it is suggested that the losses due to shrinkage and creep be assumed to be equal when the stress in the concrete adjacent to the steel is 1000 lb. per square inch.

If the ratio of the strength of the concrete at transfer to its final strength is known, its effect may be allowed for by introducing the coefficient k given in the German Specification.[1] In general, however, this ratio will not be known, and the maximum values given in the foregoing should be adopted.

For members in which the steel is post-tensioned between two and three weeks after the concrete is placed, the British draft Code specifies a maximum creep of $0{\cdot}25 \times 10^{-6}$ for a stress of 1 lb. per square inch in the concrete adjacent to the steel, and the losses due to creep are consequently 75 per cent. of those arising when the steel is pre-tensioned.

The losses given in the foregoing in the case of pre-tensioned and post-tensioned steel are based on the assumption that a minimum cube strength of 6000 lb. per square inch is obtained at the time the concrete is prestressed. When this strength is not obtained, the draft Code recommends that the losses should be increased in the ratio $\frac{6000}{C_t}$ (C_t is the cube strength when the prestress is applied), and although the draft Code recommends the application of this factor only when C_t is less than 6000 lb. per square inch, its use might also be considered for higher values of C_t. The maximum stress in the concrete when the prestress is applied and the specified cube strengths are both known to the designer, who can therefore decide whether the ratio $\frac{6000}{C_t}$ given in the British draft Code or the factor k given in the German Standard is the more appropriate. It must be borne in mind that losses due to creep are proportional to the stress in the concrete only when this does not exceed one-third of the cube strength,[1] and it is therefore preferable to allow for a stress at transfer of $0{\cdot}4\ C_t$ and not $0{\cdot}5\ C_t$ (with a maximum of 3000 lb. per square inch) as proposed in the draft Code. It is of interest that the German Standard permits maximum stresses at transfer of 2960 lb. per square inch for rectangular sections in simple bending, 3130 lb. per square inch at the corners of rectangular sections subjected to bending in two directions, and only 2840 lb. per square inch in I or T sections, compared with the single maximum value of 3000 lb. per square inch recommended in the British draft Code. On the other hand, the German Standard requires the use of concrete with a compressive strength of about 8500 lb. per square inch at 28 days (corresponding to about 6400 lb. per square inch when the prestress is applied) while the British draft Code recommends a strength of 6000 lb. per square inch when the prestress is applied. It appears desirable therefore to specify that the concrete shall have a compressive strength of $2\frac{1}{2}$ times the maximum stress in the concrete at the time when it is prestressed; the stress f_{st} in the concrete adjacent to the steel will then not exceed about one-third of the strength of the concrete (since the stress in the concrete adjacent to the steel is less than the maximum) and the

relationship between f_{st} and the losses due to creep will be linear. In assessing this stress the effect of bending stresses due to dead load at the time of prestressing may be taken into account if provision is made to ensure their presence; it may be necessary during prestressing to separate the concrete from its bottom shutter.

If it is known when the design is made that the concrete will have a high percentage of its final strength when the prestress is applied, a smaller loss due to creep may be allowed. This is the case, for example, when prestressing is carried out in two stages with a known time interval between them. Reduction factors of 0·75 for an interval of three months or 0·65 for an interval of six months appear to be safe.

The ratio of the losses due to maximum creep (on the assumption that $E_s = 29 \times 10^6$ lb. per square inch) and those due to elastic shortening for the same stress in the concrete is $\frac{9{\cdot}6}{m}$ for pre-tensioned steel and $\frac{7{\cdot}2}{m}$ for post-tensioned

TABLE I.—RELATION BETWEEN CUBE STRENGTH AND MODULAR RATIO.

APPROX. CUBE STRENGTH (lb. per sq. in.)	9500	8500	7700	6500	5600	4900	4300
MODULAR RATIO m.	4·5	4·75	5·0	5·5	6·0	6·5	7·0

steel. If the limits of m are 4·5 and 7 (see *Table* 1), the ratio varies between 1·03 and 2·13. In the U.S.A., where there are greater variations of temperature and humidity than in Great Britain, limiting ratios of 1 for very humid conditions and 3 for very dry conditions are recommended.[3]

Losses Due to Relaxation of the Steel.

Relaxation varies considerably according to the type of steel, and low losses due to relaxation should be assumed only when the steel is known to possess such properties. If the properties are not known when the design is made safe values must be allowed. In the case of German wire it is claimed that no relaxation at all occurs below the so-called " creep limit ". In the case of U.S.A. wire, a loss of 5000 lb. per square inch is usually stated to be sufficient when the initial tensioning stress does not exceed two-thirds of its ultimate tensile strength. The British draft Code tentatively recommends that a loss of 15,000 lb. per square inch be allowed for wire in the " as drawn " condition, which may be reduced to 10,000 lb. per square inch when it is straightened and subsequently heat-treated, or when an overstress of 10 per cent. of the initial tensioning stress is applied for two minutes during tensioning. It is also left to the discretion of the designer to reduce this last-mentioned allowance if he is satisfied that the properties of the steel warrant it. The use of any wire in the " as drawn " condition is not usually desirable. Losses of 15,000, 10,000 and 5000 lb. per square inch are considered in this article.

Losses Due to Friction.

Friction may be caused by variations of the profile of a duct intended to be straight, with consequent contact between the steel and the sides of the

duct, and by curvature of the steel. The prestressing force is therefore reduced when the length of the wire or bar is increased and its slope changes. The loss in a straight wire or bar depends mainly on the type of duct or sheath employed; when a flexible sheath is used the loss may also be affected by the extent to which vibration is used in placing the concrete. The loss due to curvature depends on the total angle through which the steel is curved and on the coefficient of friction between the steel and the surface along which it moves.

The loss due to accidental variations in the profile of the duct may be evaluated from the expression

$$P_o(1 - e^{-Kx}) \quad . \quad . \quad . \quad . \quad . \quad (8)$$

in which P_o is the prestressing force in the steel at the jacking end, e is the base of Napierian logarithms, K is a constant, and x is the distance from the jack to the point at which losses are required to be known. The value of K depends on the type of duct or sheath, the type of steel, and the vibration employed in placing the concrete, and the British draft Code recommends a normal value of not less than 1×10^{-3} per foot of length. When rigid ducts are used, or when special care is taken to support them firmly, K may be reduced to $0{\cdot}5 \times 10^{-3}$.

Losses due to curvature of the steel are given by

$$P_o(1 - e^{\frac{-\mu x}{R}}) \quad . \quad . \quad . \quad . \quad . \quad (9)$$

in which P_o, e, and x are as previously defined, μ is the coefficient of friction, and R is the radius of curvature of the steel. Values of μ recommended in the draft Code are given in *Table* II, together with additional data which reduce the

TABLE II.—FACTORS FOR FRICTIONAL LOSSES.

VALUES OF μ RECOMMENDED IN DRAFT CODE	
STEEL MOVING ON SMOOTH CONCRETE:	0·55
STEEL MOVING ON STEEL (FIXED TO DUCT):	0·35
STEEL MOVING ON STEEL (FIXED TO CABLE):	0·25
STEEL MOVING ON LEAD:	0·25
VALUES FOR CIRCULAR CONSTRUCTION	
STEEL MOVING ON SMOOTH CONCRETE:	0·45
STEEL MOVING ON STEEL (FIXED TO CONCRETE):	0·25
STEEL MOVING ON STEEL ROLLERS:	0·10

VALUES OF Kx OR $\frac{\mu x}{R}$	VALUES OF e^{-Kx} OR $e^{-\frac{\mu x}{R}}$
0·01	0·990
0·02	0·980
0·03	0·970
0·04	0·961
0·05	0·951
0·06	0·942
0·07	0·932
0·08	0·923
0·09	0·914
0·10	0·905
0·11	0·896
0·12	0·887
0·13	0·878
0·14	0·869
0·15	0·861
0·16	0·852
0·17	0·844
0·18	0·835
0·19	0·827
0·20	0·819

work of evaluating expressions (8) and (9). Losses due to friction may be reduced either by tensioning from both ends (when the maximum value of x is halved) or by a temporary increase in the prestressing force (*Fig.* 3). When the prestressing force is reduced to its correct value, the consequent reduction of prestress is a minimum at the point farthest from the jacks. The prestressing force shown in *Fig.* 3 is required to be constant throughout the entire length, but this need not be the case. Other methods of reducing friction have been suggested by Dr. Leonhardt, Professor Zerna, and Dr. Hahn. In Dr. Leonhardt's method, which has been used with success for the long stranded cables of the Leonhardt-Baur system, additional jacks are placed at openings formed along the cable

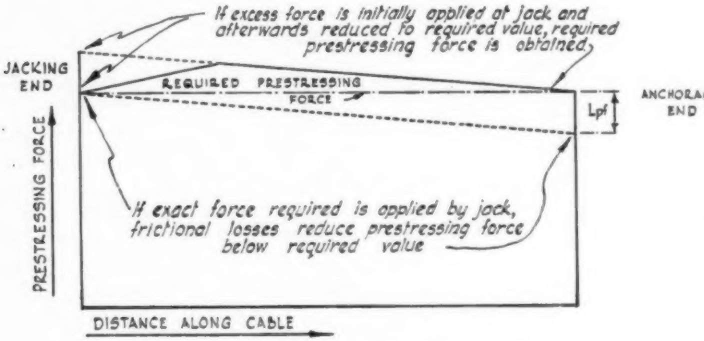


Fig. 3.—Reduction of Frictional Losses.

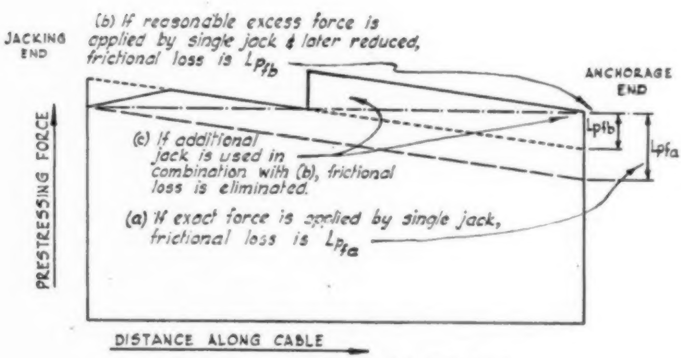


Fig. 4.—Dr. Leonhardt's Method.

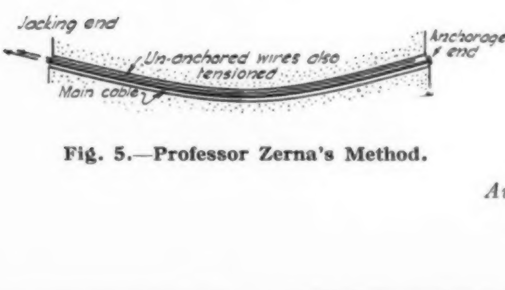


Fig. 5.—Professor Zerna's Method.

(*Fig.* 4). In Professor Zerna's method an auxiliary cable, which is not anchored, is placed between the anchored main cable and the upper side of the duct. After the main cable has been tensioned, the tension being maintained constant, the auxiliary cable is tensioned and pulled out from the duct ; as a result the original frictional losses in the main cable are offset by the friction developed by the extraction of the auxiliary cable (*Fig.* 5). Dr. Hahn suggests that the tensioned steel should be heated ; the expansion of the steel on heating and its contraction on cooling reduce the frictional losses in a manner similar to that shown in *Fig.* 3.

Losses which may occur with pre-tensioned steel, due to friction between the wires and the spacing diaphragms which are often employed, should also be considered. If they are partially or entirely prevented by tapping the diaphragms or by vibration, it is necessary to allow only for any residual losses.

When losses due to friction are allowed for in the calculations, the required prestressing force and the expected elongation should both be specified. The loss should not be computed for the greatest value of x, but for the length at the end of which the maximum effective prestress is still required. If the steel is tensioned from both ends, this is at mid-span for symmetrical loading ; in this case the elongation of the steel at prestressing should be computed for the stress $p_{i.av} = pi - \frac{1}{2}Lp_f$. If the steel is tensioned from one end the same expression applies but the loss $Lp_{f\max.}$ corresponding to $x = L_e$ is used, in place of Lp_f corresponding to $x = \frac{1}{2}L_e$; L_e is the entire length of the member, and not the clear span.

Losses Due to Other Causes.

Losses due to changes of temperature and also, in certain cases, to steam curing, may occur when pre-tensioning is employed. Losses due to changes of temperature after the concrete has hardened and bonded with the steel are negligible, since the coefficients of temperature of concrete and steel are about equal, but changes of temperature before the concrete has sufficiently hardened tend to cause movement of the steel relative to the concrete, thereby reducing or even destroying the bond between them. This must be avoided in any circumstances since, after slipping occurs, part of the prestress is retained by friction and lost when the member is loaded, so that such a structure cannot be considered as being prestressed.

Losses due to steam curing may possibly arise in a similar manner if it is applied before a certain amount of bond has developed, which usually takes place after about three or four hours. The rise in temperature of the steel may result in relative movement between the concrete and steel which would be detrimental to the development of bond, and contraction of the concrete when the temperature is lowered may reduce the stress in the steel. If steam curing is applied after this degree of bond has developed, as is usually the case, the losses are negligible.

The foregoing considerations apply only in the long-line process. If the pre-tensioning is done in separate steel moulds, the moulds expand with the wires and the losses are negligible.

In view of these possibilities, it is important when the long-line process is used to specify that, when varying temperatures are expected to occur, the tensioning process be carried out when the temperature is high, and that the concrete be quickly bonded to the tensioned steel whenever large changes of

temperature may occur. This ensures that bonding does not occur at a time when the stress in the steel is reduced. Similarly, steam curing should be delayed until after the bond has developed.

The Entire Losses.

Losses of prestress can be considered to depend on seven factors, which may be denoted by a, b, c, d, x, y, z. The factors a, b, and c depend on the conditions of casting, curing, and use; d depends on the type of duct and the curvature of the steel, if any; x represents the stress at transfer; y is the average stress in the concrete adjacent to the centroid of the steel in the tensile zone; and z represents the prestress applied by the jack.

The total loss is given by the expression

$$Lp_{total} = a + bx + cy + dz = (Lp_s + Lp_r) + b.f_{sT} + c.f_{s.av} + Lp_f \quad . \quad (10).$$

upon which Charts I and II (described later) are based. The losses Lp_s (due to shrinkage) and Lp_r (due to relaxation) are represented by the constant a; for example, the maximum loss due to shrinkage is 8700 lb. per square inch,* and a suitable value for the loss due to relaxation is 10,000 lb. per square inch. The factor b, on which the loss due to elastic shortening depends, is equal to m for pre-tensioned steel and may be $m \div 2$ for post-tensioned steel; the creep factor c, as previously shown, has maximum values of 9·6 for pre-tensioned steel and 7·2 for post-tensioned steel.

The stress $f_{s.av.}$ is the average stress in the concrete adjacent to the steel, upon which the extent of the creep depends. It should be remembered that the stress during the first month after prestressing has the same importance as the average stress during the remainder of the life of the structure. The reduction-factor of 0·9 should not be used in this connection, as $f_{s.av.}$ is assessed on this basis.

* When $E_s = 29 \times 10^6$ lb. per square inch.

(*To be continued.*)

REFERENCES.

(1) See this journal for September 1957. (2) See this journal for February 1958.

(3) Tentative Recommendations for Prestressed Concrete. A.C.I.-A.S.C.E. Joint Committee 323. "Journal of the American Concrete Institute", January 1958. Vol. 29, No. 7.

A New Masonry Cement.

A MASONRY cement is now supplied by the Cement Marketing Co., Ltd., for use in mortars for bricklaying, rendering, plastering, and non-load-bearing toppings. Tests have shown that satisfactory results are obtained with proportions of up to 6 parts by volume of well-graded sand to 1 part of this cement, and the properties of the mortar are stated to include low shrinkage and moisture movement, resistance to the effects of frost, a high degree of plasticity, workability, and cohesiveness, and a greater resistance to attack by sulphates than that of ordinary mortar. The cement is known as "Walcrete".

A Film on Bridge Construction.

A FILM showing the erection of a bridge at Cadeby Colliery, near Doncaster, has been made for McCalls Macalloy, Ltd., and may be borrowed by responsible organisations. Applications should be sent to the Company at Templeborough, Sheffield.

A Hangar at Gatwick Airport.

A HANGAR (*Fig.* 1) built on part of the site of Gatwick Airport for Transair, Ltd., is 282 ft. long and 112 ft. wide, and has a total height of 41 ft. and a clear internal height of 30 ft. On three sides there are workshops, stores, and offices, and in front of the hangar is a prestressed concrete apron.

The roof is supported by transverse triangular prestressed precast space-frames 110 ft. long (*Figs.* 2 to 6). The frames are 8 ft. 8 in. deep and have parallel members 3 ft. 4 in. apart at the top on which the roof covering is fixed. All the members in the frames were precast, and were assembled on the floor of the hangar and mortared together under their final position. The triangular cross section provided sufficient rigidity to prevent buckling when the frames were hoisted into position. At the rear of the building the lifting gear was accommodated within reinforced concrete columns (*Fig.* 7) of channel-shape cross section which support the ends of the frames. At the front of the building tubular scaffolding was used to carry the lifting gear and as a temporary support for the beams. When all

Fig. 1.—Front of Hangar : Note External Prestressing Cables of Main Beam Raised over Central Support.

Fig. 2.—Hangar during Construction.

the frames were in position they were prestressed together transversely by cables passing through the top members; the cables were composed of 0·2-in. wires which were tensioned by the P.S.C. one-wire system. Sheets of insulation board, comprising compressed straw between paper, were laid directly over the tops of the frames.

To simplify precasting and assembly, all the members of the frames have a constant cross section. The only parts of the frames in a fully prestressed state are the top members at mid-span and the diagonal and bottom members near the supports. The working stresses in the concrete were limited to 1500 lb. per square inch, and the specified minimum compressive strength of the concrete was 6000 lb. per square inch at 28 days. The frames weigh 22 lb. per square foot of the area covered, and the completed roof weighs a little less than 30 lb. per square foot.

At the rear of the building the columns are connected by precast edge-beams that also form gutters. At the front a beam 10 ft. deep (*Figs.* 8 and 9) was formed by fitting additional members between the ends of the roof frames. This beam is continuous and is supported by three prestressed columns to provide two clear openings of 140 ft.; the columns were prestressed by the Freyssinet system. This beam is subject to large shearing forces over the supports and to negligible

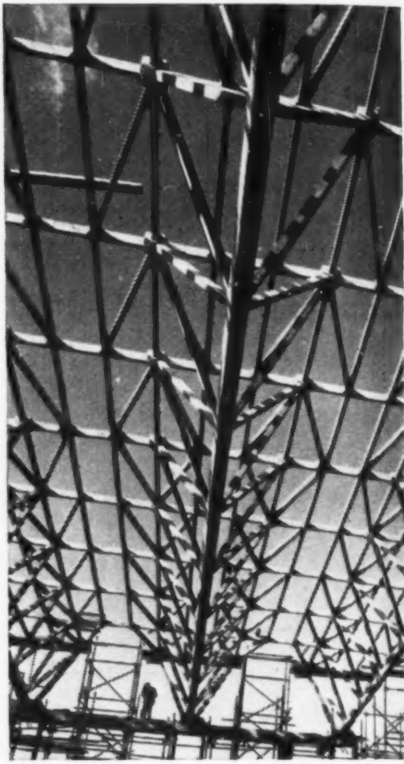


Fig. 3.—Space-frames in Position.

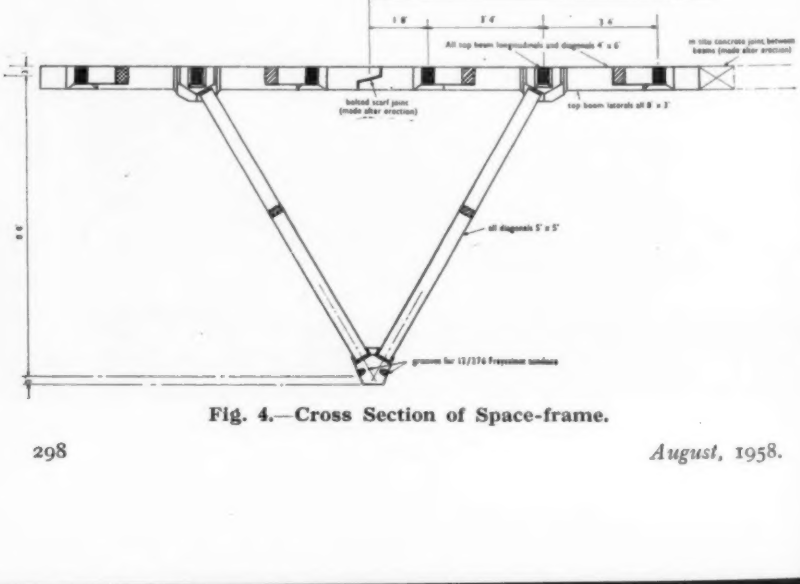


Fig. 4.—Cross Section of Space-frame.

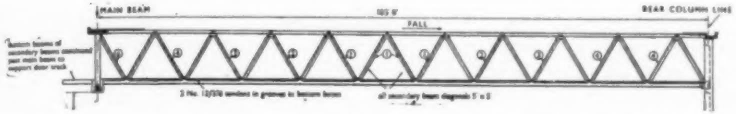


Fig. 5.—Side Elevation. Figures in circles denote number of wires in diagonal members.

forces at mid-span. Rather than make members with many different cross sections the prestress is applied by means of two external parabolic cables. The upward force applied by these cables balances the shearing force due to the dead load and greatly reduces the stresses in the diagonal members, so that only a few secondary prestressing cables were needed. Each main cable comprises four separate cables each comprising twelve 0·276-in. wires ; the cables are grouted in metal tubes. The prestress in the diagonal members was applied by the P.S.C. one-wire system while the members were on the ground.

The columns in the end walls are hinged to the secondary beams ; these columns thus carry no axial load and act solely as vertical beams. The fourth side of the hangar comprises power-operated folding and sliding doors. At the front end of each secondary beam the bottom boom projects through the triangular end-frame to support a small canopy and the door fixings.

The annexes have frames of precast reinforced concrete, aluminium window sections between the columns and, between and under the windows, ply-glass panels similar to those on three sides of the hangar.

The prestressed concrete apron measures 290 ft. by 230 ft. The slab is 5 in. thick and is laid on two layers of waxed

Fig. 6.—Junction of Bottom and Diagonal Members.

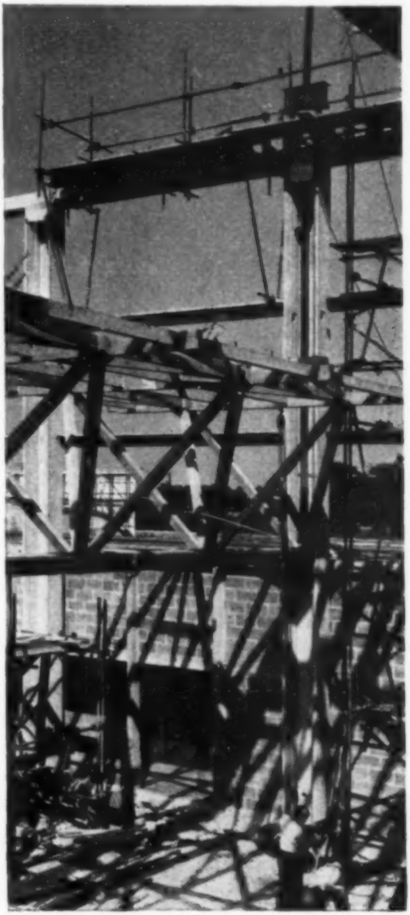


Fig. 7.—Raising a Space-frame.

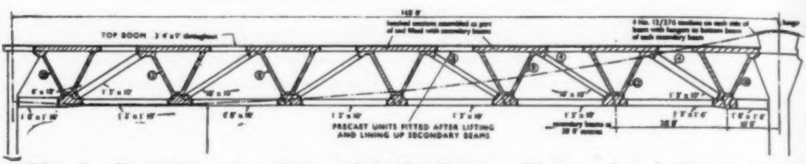


Fig. 8.—Elevation of a Span of Main Beam. Figures in circles denote number of wires in diagonal members.

paper covering 3 in. of oversite concrete. It is prestressed by Freyssinet cables, each consisting of twelve 0·2-in. wires at 3 ft. centres longitudinally and 5 ft. centres transversely. The anchors bear on reinforced concrete precast edge-beams, and concrete was later cast around the anchors. The slab was concreted in strips 15 ft. wide. One strip was cast during one day and three of the five cables were tensioned the following day. When all the strips were concreted the transverse cables were threaded through prepared holes and tensioned, after which the remaining longitudinal cables were tensioned. The cables were grouted a week after they were tensioned; the grout contained pulverised-fuel ash and was injected by a diaphragm pump.

The architects for the work were Messrs. Clive Pascall & Peter Watson. The structural frame was designed and erected by the London Ferro-Concrete Co., Ltd., in association with Messrs. A. J. & J. D. Harris, who were also the consulting engineers for the apron. Sir Alfred McAlpine & Son, Ltd., were the general contractors.

Fig. 9.—Erecting Beams between Ends of Space-frames to form Beam over Doors.

The Strength of Concrete at High Temperatures.

THE results of tests to determine the effect of high temperatures on the crushing strength of concrete are given in "Fire Research, 1956" (published by H.M.S.O. Price 4*s*.), from which the following notes are abstracted.

No significant decrease in the strength of concrete made with Portland cement and gravel aggregate was observed, either while the concrete was hot or after it had cooled, after heating at a temperature of 100 deg. C. for periods up to three months.

Table I summarises preliminary results obtained using higher temperatures. The aggregate-cement ratio for the test specimens was 4·5 by weight. The specimens tested for residual strength were allowed to cool in the laboratory and crushed fourteen days after the end of the heating period.

TABLE I.

Duration of high temperature	Percentage of strength at normal temperatures when crushed hot.	
	200 deg. C.	300 deg. C.
Less than 1 hour	95	86
1 day	94	89
7 days	82	77
28 days	78	62
	Percentage of strength at normal temperatures when crushed after cooling.	
	200 deg. C.	300 deg. C.
Less than 1 hour	75	66
1 day	65	58
7 days	55	54
28 days	52	50

A Grandstand near London.

THE reinforced concrete grandstand (*Figs.* 1 and 2) recently built for the Richmond (Surrey) Athletic Association can accommodate 1000 spectators, and provision has also been made for a royal box and ancillary accommodation. Sketch designs were prepared and approved 2½ weeks after the destruction by fire of the wooden stand in 1957, working drawings were completed two months later, tenders were received on 5 November, 1957, construction started on 1 December, 1957, and the work was completed for the Royal Horse Show in 1958. Behind the stand is a single-story structure which accommodates the tea-room and bar. Below the stand are changing rooms, kitchens, and other accommodation.

The length of the stand is 173 ft. 4 in. Ribs at 13 ft. 4 in. centres support a slab

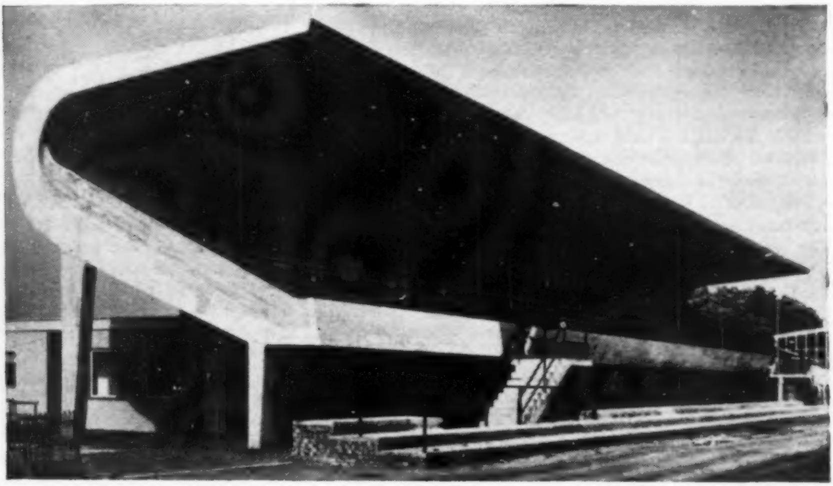


Fig. 1.

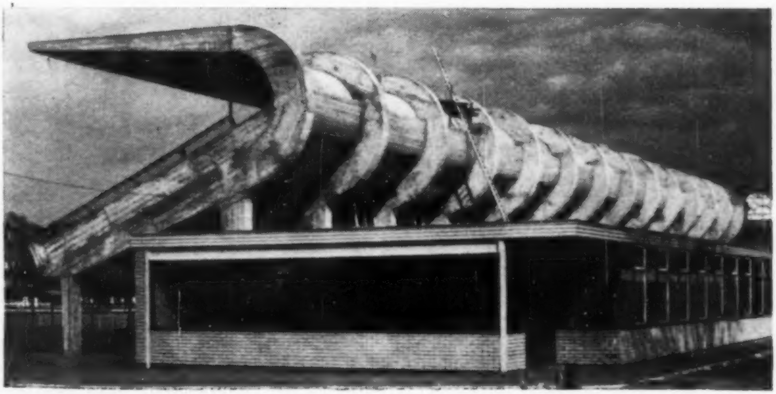


Fig. 2.

6 in. thick that carries the seats and a roof $4\frac{1}{2}$ in. thick which cantilevers a distance of 25 ft. Each rib is supported on two columns and is reinforced with high-tensile steel; the slabs are reinforced with mild steel. The seats are supported on steel brackets bolted to the fronts of the steps.

No provision is made for waterproofing the roof because it was anticipated that as the slab is in compression no leakage would occur, and tests have shown this assumption to be justified. The roof, including the beams, was concreted in two parts. Copper strips were inserted in all the construction joints.

Rainwater is carried to a gutter at the back of the roof from which it passes into polythene pipes of 3 in. diameter in the centre of the lower part of the ribs and into the supporting columns and thence into a horizontal cast-iron pipe, 6 in. in diameter, in which the rainwater from the roof over the tea-room and bar is also collected and transferred to each end of the structure. No down-pipes are therefore visible.

The proportions of the concrete were $1 : 1\frac{1}{2} : 3$, with rapid-hardening Portland cement. The surface of the concrete is as left from the shuttering. Wrought boards were used for the ribs and columns, plywood lining for the soffits of the floor and roof slabs, and narrow sawn boards for the lower rear curve and the soffit of the floor of the stand.

The single-story building behind the stand has a steel frame, and a detail of the connection between the steelwork and the reinforced concrete columns is shown in *Fig.* 3. Four vertical slots were drilled through the plates at the ends of the roof beams. Similar plates, also drilled and with hexagonal nuts welded on the back to receive the initial fixing bolts, were cast in the faces of the columns. Four reinforcement bars were also welded to the back of each plate to improve the bond between the plate and the concrete. The steel joists were bolted to the column-plate and the two plates were welded together. The fixing bolts were then removed, and the slots filled with metal.

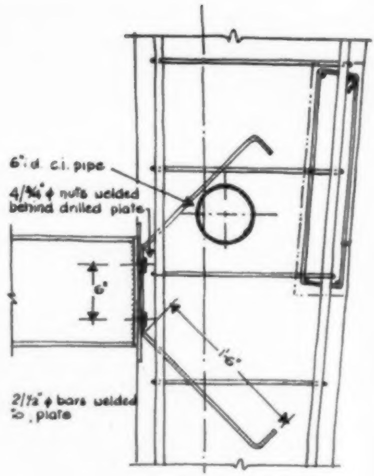


Fig. 3.

The architects were Messrs. Manning & Clamp, the consulting engineers Messrs. Jenkins & Potter, and the general contractors Messrs. Percy Bilton, Ltd. The steel frame was erected by Messrs. Mackey Bowley Co., Ltd.

Vaulted Slabs for New Airport Building.

THE illustration is of a model of a building to be built at New York International Airport for Trans-World Air Lines. The structure will comprise four vaulted concrete slabs supported on concrete frames, and the walls will be of glass. The work was scheduled to start in April 1958, and to be completed by January 1960; the estimated cost is $12,000,000. The architects are Messrs. Eero Saarinen & Associates, and the consulting engineers are Messrs. Ammann & Whitney.

Bond between Concrete and Steel.

A BULLETIN issued by the Iowa State College (Engineering Report No. 26, Iowa State College, Ames, Iowa, U.S.A.) comprises papers by H. J. Gilkey on " Hardened Concrete ; Bond with Reinforcing Steel ", by H. J. Gilkey, S. J. Chamberlin, and R. W. Beal on " Bond between Concrete and Steel", and by S. J. Chamberlin on " Spacing of Spliced Bars in Tension Pull-out Specimens " and " Spacing of Reinforcement in Beams ". The papers describe tests made by the authors, and the results are summarised as follows.

Contrary to accepted belief, bond resistance is not proportional to the compressive strength of standard-cured concrete, there being some increase in bond but a reduction in the ratio of bond resistance to ultimate compressive strength as the strength of the concrete increases, especially in the case of higher strengths.

Contrary to design assumptions, the bond developed by added length of embedment of bar is not proportional to the added length of the embedment. The shorter the embedment the greater is the average unit bond stress that can be developed by a plain bar.

The development of bond resistance is progressive along the bar, maximum intensity of resistance being attained and passed in each section successively as increments of load on the bar increase. The smoother the bar the shorter is the length along which the maximum intensity of resistance can be effective.

For a short embedment the total pull developed prior to " first slip " is largely a function of the peak bond intensity that can be developed. As length of embedment increases, the total pull developed by a plain bar prior to " first slip " increases, but the average unit bond decreases, approaching, as a limiting value, the bond corresponding to " residual drag ".

After any point along an embedded plain bar has attained and passed its maximum intensity of resistance, this intensity is reduced more or less gradually to a nearly constant value approaching that of the residual drag. The smoother the bar the less the residual drag. For ordinary hot-rolled material the residual drag approximates half of the maximum resistance developed.

Slip at the loaded end of a pull-out specimen starts with the first application of load and continues as load is added. The slip at the loaded end prior to " first slip " at the unloaded end is about proportional to the length of the embedment. After " first slip " occurs movements of both ends of the bar are about equal. The load corresponding to " first slip " for plain hot-rolled bars in vertically-cast specimens is 80 to 85 per cent. of the ultimate load. For the same bars in horizontally-cast specimens the load at " first slip " is 90 to 95 per cent. of the ultimate load. Load at " first slip " for plain hot-rolled rusted bars in vertically-cast specimens is 80 to 90 per cent. of the ultimate load. For relatively smooth high-strength alloy-steel bars " first slip " occurs at the ultimate load, as it does also for cold-drawn wire and for bars coated with a light layer of grease.

Horizontally-cast plain-bar pull-out specimens apparently develop only about 70, 65, and 45 per cent. of the resistance developed by similar vertically-cast specimens at " first slip ", ultimate load, and residual drag respectively for plastic mixtures with different cements. For similar mixtures and specimens made with different cements the bond strength varies less than does the compressive strength of the mixture. The cements producing the strongest concretes produce the strongest bond, but those producing the weakest concretes have the highest ratio of bond to compressive strength.

The greater the tensile stress in the steel at " first slip ", the less is the increase of the ultimate load over the load at " first slip ". Little total bond resistance beyond that necessary to develop the yield-point stress in the steel can be developed by a plain bar.

Deformed bars perform in the same manner as do plain-bar specimens until slip at the loaded end is sufficient to bring some of the lugs into bearing. Short deformed bars register about the same load at " first slip " as plain bars, but the longer deformed bars have higher average bond stresses at " first slip ". The load on a deformed-bar pull-out specimen continues to increase after " first slip " (with considerable additional slip) until the concrete splits, the lugs plough through the concrete, or the bar fails in tension.

Splitting or ploughing may occur at stresses in the bar either greater or less than its yield-point strength.

Relatively little of the excess resistance of a deformed bar is available against bond-failure in a beam because the slip required to bring lugs into contact exceeds that required to produce failure or to damage a flexural member severely. However, the use of deformed bars at bond stresses about 20 per cent. in excess of those for plain bars may be justified on the basis of some added resistance for moderately long embedments and a considerable added margin of safety against complete collapse of a member. Obviously, there is a need for better types of deformed bars having a fine-textured and non-wedging roughness.

The stronger the concrete the greater is the resistance developed in a deformed-bar specimen before splitting occurs. Added depth of cover between 1 in. and 2 in. increases the ultimate resistance of deformed bars appreciably and the resistance at "first slip" slightly. There is nothing to indicate that increased depth of cover can be justified as a device for preventing or deferring splitting due to the wedging action of the lugs. Whether a deformed bar ploughs through the concrete or splits it seems to depend on the depth of cover and to be independent of the strength of the concrete within the range of mixtures investigated (3000 to 5000 lb. per square inch compressive strength).

A vertically-cast pull-out specimen appears to be satisfactory. It provides results which may be either absolute or relative, depending upon such important variables as differences in the ratio of length to depth, the method of placing the concrete, and the orientation of members at the time of casting.

Whether the steel happens to be surrounded by concrete in tension as in a beam, or by concrete in compression as in a pull-out specimen, seems to have no significant effect either on the nature or the magnitude of the bond developed.

A slip at the free end of less than 0·01 in. appears to mark the limit of usefulness of a bar in a beam.

Deep flaky rust on bars lowers the bond, but wiping the loosest rust off firmly with canvas produces a surface that will develop a bond equal to or greater than that which the bar would have developed in the unrusted condition. The loose powdery rust which appears on bars during the first few weeks of ordinary exposure has no significant effect on the bond properties of the bars.

Clean drawn wire develops a resistance only slightly greater than that developed by greased plain bars, which develop much less bond at all stages of test than do ungreased plain bars. Greasing deformed bars decreases the load at which "first slip" occurs, but has little apparent effect upon the loads developed after the lugs are brought into bearing.

Variations in age and type of curing seem to alter bond resistance much less than they alter the compressive strength of the concrete.

Loading through springs to simulate a load free to follow the movement of the bar has little effect on the resistance developed by deformed bars, but in the case of plain bars such loading causes a sudden excessive slip at about the load which is normally recorded as that at "first slip".

A deformed-bar pull-out specimen can sustain a load greater than that at "first slip" for several weeks without evidence of distress or reduction in the load which the specimen can resist.

Autogenous healing in bond occurs under the same conditions as it occurs in tension or compression. Undamaged specimens subjected to moist storage after a bond test beyond ultimate load may show re-test bond strengths equal to or exceeding those of the earlier test. There is no bond-recovery for specimens which have not had access to moisture during the period between tests.

Vertically-cast beams have about the same margin of safety against failure by bond and diagonal tension as against tensile failure of the steel. The margin of safety for bond and diagonal tension in horizontally-cast members is at least 25 per cent. less than that for tension in the steel.

The use of smooth bars, such as cold-drawn wire or polished or cold-rolled bars, at the same bond stresses as hot-rolled bars provides an undesirably low margin of safety, as does the use of long embedments of any type of plain bar.

An Office Building at Cambridge.

A FOUR-STORY office building for the Eastern Region of British Railways at Cambridge is shown in *Fig.* 1. The ground floor has a reinforced concrete frame and the upper stories were built on the "Intergrid" system of precast prestressed components as shown diagrammatically in *Fig.* 2. The structure comprises columns, boundary beams, primary and secondary beams, and floor and roof slabs. When the slabs are grouted in position they act with the beams to form a monolithic plate. *Fig.* 3 shows the underside of a completed floor, before the erection of a ceiling.

At the level of the ground floor the

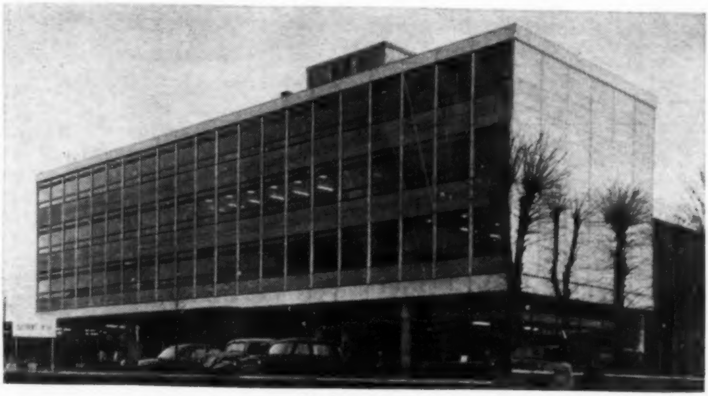


Fig. 1.

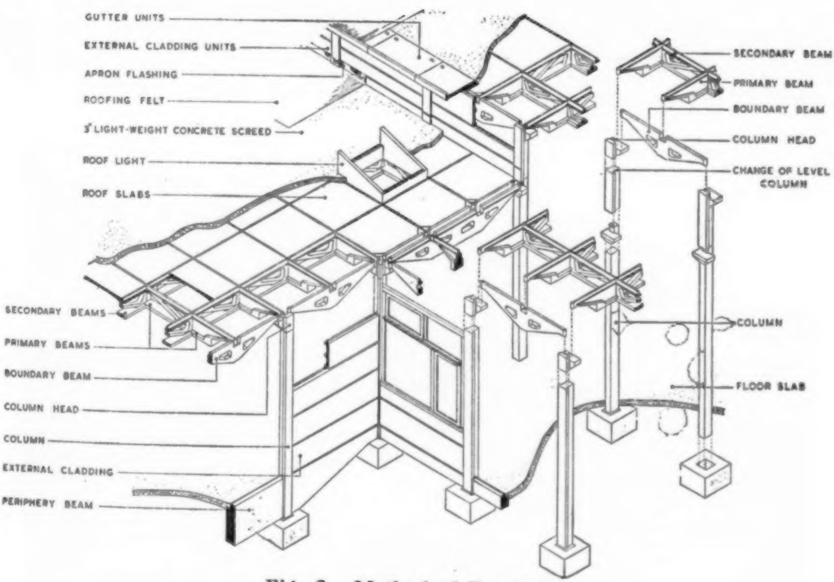


Fig. 2.—Method of Erection.

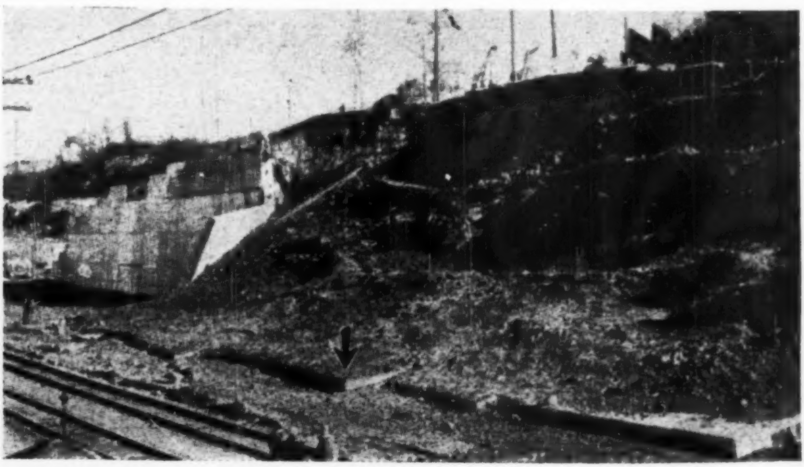
Fig. 3.—Underside of Floor.

external walls are finished with grey-brown facing bricks; above this level granite-faced precast slabs are used. The whole of the work was completed in nine months.

Mr. H. H. Powell is the Regional Architect and Mr. A. K. Terris the Chief Civil Engineer of British Railways (Eastern Region). The consulting engineers for the "Intergrid" system are the Pre-Stressed Concrete Co., Ltd., and the contractors were Messrs. Gilbert-Ash, Ltd.

Failure of a Retaining Wall.

THE collapse of part of a new retaining wall at Portland, Oregon, U.S.A., is described in a recent number of "Engineering News-Record". The wall was 70 ft. high and formed part of a widening scheme at a railway goods yard; the arrow on the illustration shows the proposed position of the new track. A section 310 ft. long overturned. The wall was 7 ft. thick at the base, tapering to 18 in. at the top. The reason for the failure is being investigated.

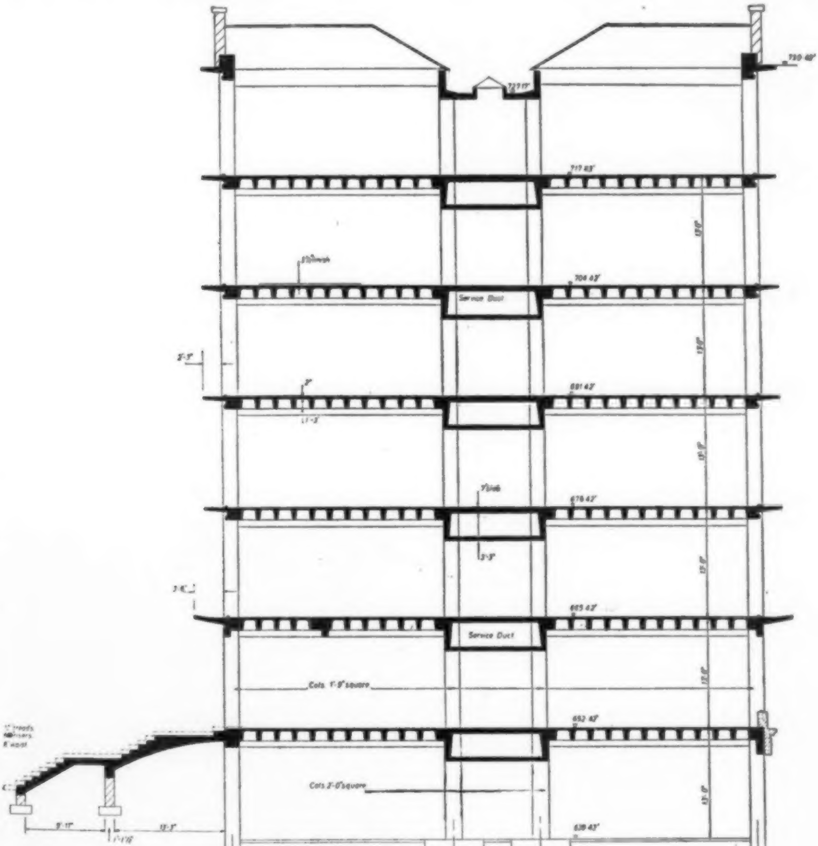


Extension of a Technical College.

THE main part of the extension of the Halifax Technical College (*Figs.* 1 and 2) is a multiple-story building 240 ft. long and 62 ft. wide, comprising six stories and a basement under part of the structure (*Fig.* 3). The building is a reinforced concrete framed structure with columns at 24 ft. centres longitudinally and generally at 25 ft. 9 in. and 10 ft. 3 in. centres transversely.

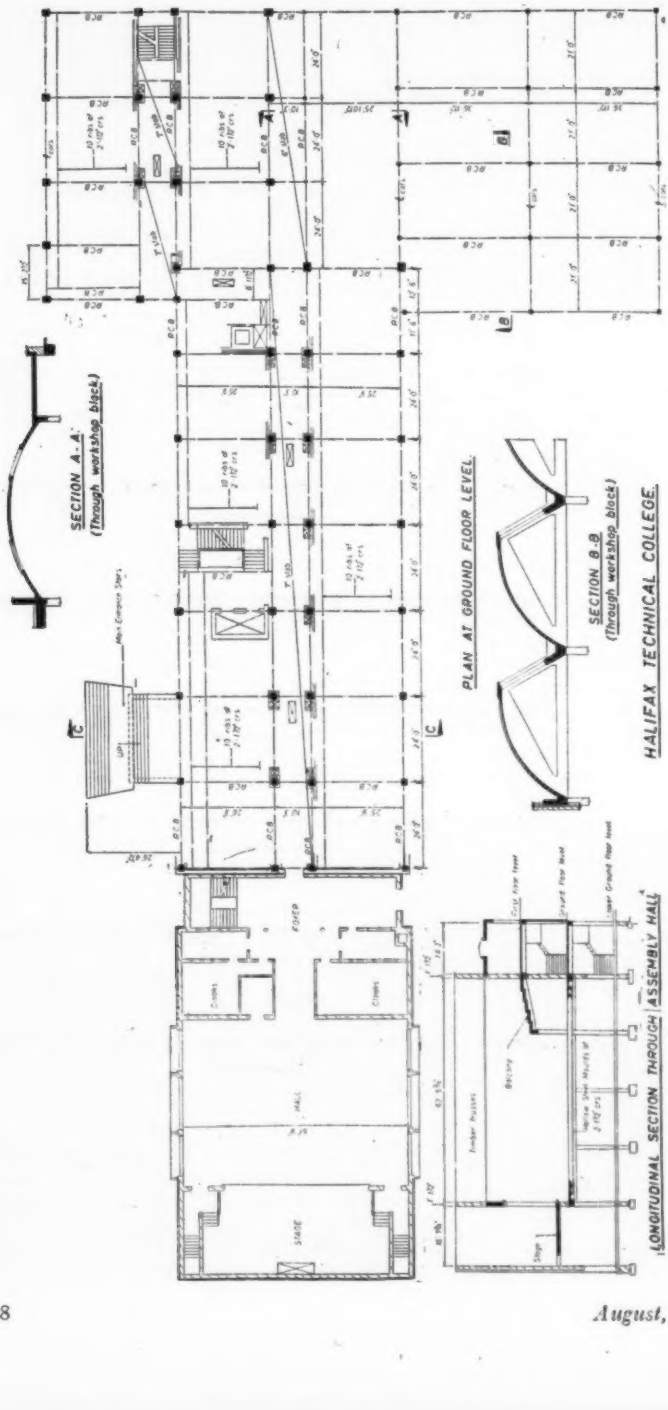
The column bases are on rock with a bearing strength of about 10 tons per square foot. The columns are 2 ft. square from the base to ground-floor level, above which they are 1 ft. 9 in. square.

The ground floor is 13 ft. above the lower ground floor. The suspended floors are of hollow steel-mould construction in the larger bays, the ribs extending the length of the building and being supported by beams at 24 ft. centres. The ribs are 1 ft. 3 in. deep by 5 in. wide and there is



CROSS-SECTION C-C

Fig. 1.



HALIFAX TECHNICAL COLLEGE.

Fig. 2.

a 2-in. topping making a total depth of 1 ft. 5 in. In addition there is a 3½-in. sound-resisting finish. The floor of the central corridor, 10 ft. 3 in. wide and 7 in. thick, was cast in place; below it is a service duct 2 ft. 10 in. deep and a lower slab 5 in. thick. The walls of the duct form beams to carry the top and bottom slabs and the brick walls of the corridors. The class-room floors were designed generally for a load of 80 lb. per square foot and the corridors and stairs for a load of 100 lb. per square foot. Vertical service ducts were formed at one side of most of the interior columns to enclose rainwater pipes, heating pipes, electricity supply, and so on.

Where the right-angle turn occurs in the horizontal ducts a beam was used with large octagonal holes to receive the services and to enable the heating pipes from the boiler in the basement to enter the duct; this beam is designed as a Vierendeel girder.

From the first floor upwards concrete balconies cast in place (*Fig.* 4) extend nearly the full length of the building, with precast slabs projecting at the centres of the columns. The general exterior finish is ashlar. The roof is a light steel frame carried on reinforced concrete beams, with the exception of the roof over the corridor, which is a flat slab 6 in. thick.

Adjoining the main building are two smaller structures (*Fig.* 3) for workshops and the assembly hall. The workshop

Fig. 3.

Fig. 4.

is a single-story framed building 84 ft. long by 72 ft. wide. Over six bays there are north lights 3½ in. thick designed as "shells." The valley beams are about 3 ft. deep in two spans of 36 ft. The end and central slabs are 9 in. thick and span 21 ft. The remainder of the roof comprises a slab 10 ft. 3 in. wide by 6 in. thick and a shell 3 in. thick by 72 ft. long with a chord of 26 ft.; there are roof lights on both sides of the shell.

The assembly hall is 95 ft. long by 64 ft. wide. The columns are on a grid of 16 ft. by 20 ft. The beams which support the floor are in the shorter direction; they are 1 ft. 3 in. thick, in three continuous spans of 20 ft. each. The balcony is supported on beams at about 8 ft. centres; the slab is 6 in. thick.

The architects are Mr. R. H. Pickles in association with Messrs. Clement Williams & Sons. The contractors for the main building and workshop were Messrs. L. & W. Morrell, Ltd.; Messrs. W. Parker, Ltd., were the contractors for the assembly hall. The British Reinforced Concrete Engineering Co., Ltd., designed and supplied the reinforcement.

Book Reviews.

"Stützmomenten-Einflussfelder durchlaufender Platten." By Günter Hoeland. (Berlin: Springer Verlag. Price 37.50 D.M.)

This book gives "influence surfaces" for the determination of the support moments of continuous slabs, and contour lines are given for any load in any position on a slab. The cases considered in 78 tables and diagrams include slabs freely-supported and with fixed supports clamped along the edges, and with free supports in various combinations for span ratios of 1·0, 0·75, and 0·5. The elastic yield of supporting beams is considered for various ratios of the stiffness of the slab to the stiffness of the beam. The theory of influence surfaces, which is considered to be immaterial to the application of the diagrams, is briefly mentioned, and an example explains the method of evaluating the ordinates of an influence surface by means of Simpson's rule.

Books Received.

"Tehdasrakennusten Hankintaja Pitokustannuksista Muodon ja Koon Funktiona", by O. Gripenberg.

"Analys av Kostnaderna för Några Flervånings-Bostadshus I Norden", by O. Gripenberg.

"Uusi Saunakiuastyyppi", by O. Vuorelainen.

"Lämmönsiirtoaineista", by O. Vuorelainen.

[The foregoing are printed in the Finnish language.]

"Über die Knickung und Tragfähigkeit eines Exzentrisch Gedrückten Pfeilers Ohne Zugfestigkeit", by Kyösti Angervo.

"Erweiterung der Theorie der Biegung eines Pfeilers Ohne Zugfestigkeit und Ihre Anwendung zur Berechnung von Rahmentragwerken mit Unbewehrten Stielen", by Kyösti Angervo and A. I. Putkonen.

[The foregoing are printed in the German language with short summaries in English.]

"Laastiksymyksestä", by T. Karttunen and T. Sneck.

"Linolipäällysteisiin ja Niiden Aluslattioihin Liittyvistä Vaurioista", by Risto Ruso, Tenho Sneck and Viljo Vartiainen.

"Suomen Ilmaston Pääpiirteet Erityisesti Talvirakentamista Silmälläpitäen", by Seppo Huovila and S. E. Pihlajavaara.

"Teräsbetonirakenteiden Murtolujuus ja Sitkeys", by Herman Parland.

[The foregoing are printed in the Finnish language with short summaries in English.]

"Thermal Conductivities of Building Materials in Dwelling Construction", by T. T. Tuomola and R. R. Russo. (Printed in the English language.)

Published by Staatliche Technische Furschungsanstalt, Helsinki. No prices stated.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", July–August, 1908.*

The Late Mr. L. G. Mouchel.

It is with great regret that we record the death of Mr. L. G. Mouchel, who for several years represented M. Hennebique, of Paris, in this country, and in his capacity as representative of that well-known firm and as a civil engineer of high inventive faculties did much to introduce and later on to develop reinforced concrete . . . Handicapped by encountering prejudices against reinforced concrete and a conservatism that did little credit to our engineering profession, he nevertheless, by sheer pertinacity and the manner in which he inspired confidence, obtained the ear of professional men of influence who adopted the system of construction he represented, and arranged with contractors of the highest position to become licensees for the execution of work that was entirely new to them.

* "Concrete and Constructional Engineering" was published in alternate months until September, 1909.

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